

CANADA - NWT

MINERAL DEVELOPMENT AGREEMENT

**POST PILLAR RECOVERY STRATEGIES AT
NANISIVIK MINE, NANISIVIK, NORTHWEST TERRITORIES
PHASE 1: GROUND CONTROL INSTRUMENTATION
AND STRESS DETERMINATIONS**



**POST PILLAR RECOVERY STRATEGIES AT
NANISIVIK MINE, NANISIVIK, NORTHWEST TERRITORIES
PHASE 1: GROUND CONTROL INSTRUMENTATION
AND STRESS DETERMINATIONS**

B. ARJANG AND G. HERGET

MRL89-147(TR)

NOVEMBER 1989

This document is an unedited
internal report prepared primarily
for discussion and internal
use only. It does not
represent a final expression of
the opinion of the Canada Centre
for Mineral and Energy Technology
(CANMET).

Ce document est un rapport
provisoire non-révisé et rédigé
principalement pour fin de
discussion et de documentation
interne. Il ne représente nullement
l'expression définitive de
l'opinion du Centre canadien de
la technologie des minéraux et de
l'énergie (CANMET).

This project was funded in part by the Canada/NWT Mineral Development Agreement and was conducted under the auspices of CANMET's Mining Research Laboratories.

CANMET Project No. 144610-01-4

CANMET Scientific Authority: Dr. Somchet Vongpaisal

MICROMEDIA

46pp

2



Energy Mines and
Resources Canada

Energie, Mines et
Ressources Canada

CANMET

Canada Centre
for Mineral
and Energy
Technology

Centre canadien
de la technologie
des minéraux
et de l'énergie

POST PILLAR RECOVERY STRATEGIES AT NANISIVIK MINE, NORTHWEST TERRITORIES PHASE 1: GROUND CONTROL INSTRUMENTATION AND STRESS DETERMINATIONS

B. ARJANG AND G. HERGET

ELLIOT LAKE LABORATORY

NOVEMBER 1989

POST PILLAR RECOVERY STRATEGIES AT NANISIVIK MINE, NORTHWEST TERRITORIES
 PHASE 1: GROUND CONTROL INSTRUMENTATION AND STRESS DETERMINATIONS

B. Arjang* and G. Herget**

ABSTRACT

The Nanisivik Mine produces zinc and some lead ore from a near surface underground mine in permafrost. The potential exists to increase ore extraction from the present 80% by pillar recovery. Determinations of ground stresses, rock strength, deformation parameters and video inspection of roof competence were carried out to assess present and future excavation stability. In addition, ground movement monitors and CANMET strain monitors were installed at a selected site to determine ground reactions during pillar recovery.

Ground stress determinations by overcoring CSIR triaxial strain cells were successfully carried out in permafrost (-15°C). For the East portal of the mine, determined pre-mining vertical stress and stresses parallel and perpendicular to the horizontal orebody were as follows:

Vertical stress : 0.028 MPa/m

Stresses perpendicular to the orebody : 3.2 x vertical stress

Stresses parallel to the orebody : 1.4 x vertical stress

Deformation moduli, determined from the dilatometer tests and the geomechanics rock mass rating were in the range of 50-60 GPa for dolomite, and 20-30 GPa for the orebody.

Based on the results, a numerical model will be developed to analyze the stability of underground excavations at different phases of selective pillar removal.

Key words: Arctic; Permafrost; Post pillar; Ground stress; Roof stability; Monitoring.

*Research Scientist, Elliot Lake Laboratory, CANMET, Energy, Mines and Resources Canada, Elliot Lake, Ontario.

**Research Scientist, Canadian Mine Technology Laboratory, CANMET, Energy, Mines and Resources, Ottawa, Ontario.

CONTENTS

	<u>Page</u>
ABSTRACT	i
RESUME	ii
INTRODUCTION	1
GEOLOGICAL SETTING AND MINING OPERATION	1
TEST AND INSTRUMENTATION SITE	3
GROUND STRESS DETERMINATIONS	3
LABORATORY TESTING	6
RESULTS OF GROUND STRESS DETERMINATIONS	6
DETERMINATION OF THE IN SITU DEFORMATION MODULUS	8
STRAIN MONITORING	9
GROUND MOVEMENT MONITORING	10
VIDEO PROBE INSPECTION OF ROOF HOLES	11
CONCLUSIONS	11
ACKNOWLEDGEMENT	12
REFERENCES	12

TABLES

No.

1. Diametric deformations from USBM meter measurements	14
2. Elastic strain recovery from CSIR triaxial strain cell measurements	15
3. Determination of Elastic Modulus (E), Poisson's Ratio (ν), and Uniaxial Compressive Strength (UCS) from compression tests	16
4. Magnitudes and directions of secondary principal stresses from USBM meter measurements	17
5. Stress components, magnitudes and orientations of the principal stresses from CSIR triaxial strain cell measurements	18
6. Insitu deformation moduli from dilatometer measurements	19
7. Location data and calibration factors for strain monitors	20
8. Location and initial data for ground movement monitors	21

FIGURES

1. Location of Nanisivik Mines Ltd.....	22
2. Outline of the main orebody	23
3. Overview of test and instrumentation site	24
4. Overcoring gauge responses from USBM-meter measurements, NML-A01 and NML-A02	25

5. Overcoring gauge responses from USBM-meter measurements, NML-A03 and NML-A04	26
6. Overcoring gauge responses from USBM-meter measurements, NML-P02 ..	27
7. Triaxial strain cell gauge response and linearity in biaxial compression, test NML-A06	27
8. Inside view of EX-hole containing a bonded triaxial strain cell ...	28
9. Post-failure appearance of overcored specimen, test NML-A05	28
10. Post-failure appearance of 53 mm diameter core specimen of sulphide ore from Site A	29
11. Post-failure appearance of a 137 mm diameter core specimen of dolomite from Site B	29
12. Orientation of the principal compressive stresses from Mine Grid north (=Geographic north) in an equal area net, lower hemisphere projection	30
13. Schematic of the PROBEX-1 Dilatometer system	31
14. Plot of the in situ deformation modulus measurements, Site A	32
15. Plot of the in situ deformation modulus measurements, Site C	33
16. Typical calibration cycles of the dilatometer system obtained underground (Site A)	34
17. Relationship between in situ modulus and rock mass rating, plots represent moduli determined in Tables 3 and 6	35
18. Calibration curves for strain monitors SMS-121 and SMS 122	36
19. Calibration curves for strain monitors SMS 123 and SMS 124	37
20. CANMET strain monitoring system	38
21. Ground movement monitor, GMM	39
22. Typical calibration plot of a ground movement monitor	40
23. Video probe 2000 - system components	41

INTRODUCTION

The Nanisivik Mine is located at the northern tip of Baffin Island and produces zinc and some lead from a near surface underground mine in permafrost. The potential exists to boost ore recovery from the present 80% by pillar recovery. Funding under the NWT Mineral Development Agreement was made available to Strathcona Mineral Services Ltd. to study the ground control problems which might arise during pillar recovery. Nanisivik Mines Ltd. subcontracted the rock mechanics program of the study to the Canadian Mine Technology Laboratory (CMTL) of MRL, CANMET, Ottawa (1).

The objective of the rock mechanics investigation is to achieve pillar recovery without serious deterioration occurring in roof and wall strata. The program has four elements: field investigations, laboratory testing, analysis of excavation stability and monitoring.

In the first phase, field investigations and laboratory testing were conducted. These included ground stress determinations, in situ rock deformation modulus measurements, video inspection of roof competence, installation of strain monitoring and ground movement monitoring systems and core sampling for laboratory testing at selected test sites. The information obtained will provide input to evaluate mine stability for different stages of pillar recovery with the aid of numerical analysis to be conducted by CMTL, CANMET.

Pillar recovery is planned to start after the instrumentation for ground control monitoring is completed and a numerical model of excavation stability has been developed. Monitoring of stability conditions during pillar recovery will be carried out by ground control staff of the Nanisivik Mine.

GEOLOGICAL SETTING AND MINING OPERATION

The Nanisivik Mine is situated at the northern extremity of Baffin Island, Northwest Territories, as shown in Figure 1. The Proterozoic

zinc/lead deposit consists of massive sulphides within dolomite of Helikian age. The easterly trending main orebody is up to 20 m thick, 50-120 m wide and extends on strike to about 3000 m. Overburden up to 130 m covers the horizontal orebody. No significant displacement of the orebody is known as a result of faulting. Measurements in the dolomite strata, the host rock, indicates a dominant NW/SE and a less dominant NE/SW near vertical joint set (2). The roof (back) consists of three types of rock: dolomite (massive or bedded), sulphides and shale.

The orebody is about 300 m above sea level within the continuous permafrost zone. The permafrost extends to a depth of about 600 m below surface with steady rock temperatures of -10 to -12°C. The outline of the main orebody is shown in Figure 2.

Ground conditions and permafrost permit primary room-and-pillar mining with an ore extraction of up to 80%. The ultimate extraction rate after the planned pillar recovery is estimated to reach over 90%. Pillars are spaced approximately on a 25 m x 25 m pattern with a cross-section of 7 m x 7 m, and heights up to 20 m. The roof within the dolomite rock is generally stable and artificial support is not required.

The current mine production is 2200 t/day. About 60% of the ore is mined in the main zone, and 40% comes from separate mining blocks called 'satellite zones'. Access to the main orebody is through two portals cut into the mountainside on the east and west ends of the orebody. Trackless mining equipment is used to haul the ore to the ore pass in the centre of the western zone. From the crusher a conveyor belt transports the ore to the mill for processing. It is intended to carry out the pillar recovery from the east end of the orebody by retreat mining and maintain stability for east-west access during the operation.

TEST AND INSTRUMENTATION SITE

The test site is located close to the east portal of the mine. It covers an area of about 8500 m² with the roof up to 25 m high. The maximum roof span is approximately 75 m and the supporting pillars of varying geometry are spaced at about 20 m. The roof and south abutment consist of various types of dolomite, and the north abutment is mainly sulphide rock with dolomite breccia and quartz stringers. The maximum cover at the site is about 40 m. Ambient underground temperature during the field investigation varied between -12 to -18°C with relative humidity of 45 to 60%.

Figure 3 shows an overview of test and instrumentation sites. Ground stress determination (GSD), in situ deformation modulus measurement (DMM), installation of strain monitoring system (SMS), ground movement monitor (GMM), and video borehole inspection (VBI) were carried out in the following boreholes drilled into the roof, pillars and abutments:

- Two - horizontal 6 inch holes (dia. 153 mm), site B; GSD, SMS
- One - horizontal 6 inch hole (dia. 153 mm), Pillar P37.10/2; GSD, SMS
- Two - horizontal NQ holes (dia. 76 mm), sites A and C; DMM
- Ten - vertical BQ holes (dia. 60 mm), roof; VBI, GMM
- Four - horizontal EX-holes (dia. 38 mm), pillars: P37.11/2 and P37.09/3; GMM

GROUND STRESS DETERMINATIONS

Ground stress determinations were carried out by overcoring USBM meter deformation gauges and triaxial CSIR strain cells. Details of testing procedures and instrument specifications are given in references (3-5). Measurements were taken in 153 mm diameter horizontal holes which yielded overcored test specimens of 137 mm diameter. For diamond drilling in sub-zero temperatures a brine solution consisting of 22% calcium chloride was used. This solution caused severe corrosion on the instruments and equipment used underground.

The test hole in pillar P37.10/2 intersected coarse massive sulphide ore with carbonate stringers containing numerous ice-filled vugs and cavities. One triaxial and three biaxial overcoring measurements were attempted at a depth of 4.0 to 5.5 m from the collar. The triaxial strain cell failed to bond with Philips strain gauge cement (PR9244/04) at -18°C .

Biaxial tests using borehole deformation gauges, which do not require gauge cementing, were then carried out. Only one overcoring measurement provided stable readings and acceptable results. The presence of ice-filled vugs and cavities in the weak sulphide rock of the test hole resulted in poor contact between the buttons of the USBM meter gauge and the wall of the EX-pilot hole. Because of insufficient contact, creep and/or rotation of the gauge in the hole was encountered during the overcoring process. The stress determination hole in the pillar was abandoned at a depth of 5.80 m.

Ground stress determinations in the south abutment (Site B) were carried out in two boreholes in a massive fine-grained dolomite rock containing occasional vugs and calcite coated fractures. Some vertical and sub-horizontal discontinuities intersected the stress determination holes. Four biaxial overcoring measurements were conducted at depths of 3.45 to 6.30 m in the first hole. All four tests were successful.

Table 1 provides a summary of deformation readings and field data from biaxial USBM meter measurements in pillar and south abutment locations. Overcoring gauge responses obtained from USBM meters are plotted in Figures 4, 5 and 6.

Triaxial overcoring measurements in the south abutment were carried out at a depth of 6.75 m and 3.40 m in test holes No. 1 and No. 2, respectively. Test hole No. 1 at a depth of 7.0 m had to be abandoned because of borehole deviation. The second test hole was drilled parallel to the first one at a distance of 0.6 m. Strain recovery data from the triaxial measurements are given in Table 2.

In a permafrost environment, successful bonding of strain cells in the pilot hole was achieved at Site B using two component gauge adhesive cement (Hoettinger x60, No. 214.00-2005). Prior to the final installation of strain cells, several experiments with gauge adhesive cement were attempted at the test site. Different proportions of two component cement were mixed at an ambient temperature of -12 to -14°C and a relative humidity of 60 to 65%. A mixture of two measuring cups of cement (component A) and 13 drops of hardener (component B), having a pot life of four minutes, cured after 15 minutes. An identical mixture was then prepared and applied to the surface of the strain cell rosettes and installed in the pre-cleaned EX-hole of an overcored specimen obtained from USBM meter test. The strain cell bonded after a curing time of 25 minutes. The cemented cell was retrieved by applying force. The bonding experiments proved to be successful.

The test location in the EX-pilot hole was cleaned and dried using alcohol and a generous amount of acetone. This process was repeated several times until the hole was suitably clean and dry for installation of the strain cell. To repel moisture from the surface of the pilot hole, the acetone soaked gauze on the cleaning tool was left at the test location for about 10 to 15 minutes. Installation of the cell followed immediately after the removal of the cleaning tool. Overcoring commenced four hours (Test NML-A05), and one hour (Test NML-A06) after the strain cells were installed. Complete sets of stable readings were obtained from test NML-A05. After overcoring, test NML-A06 indicated unstable readings on gauges No. 9, 10 and 11. Strain readings from these gauges were excluded from stress tensor calculations. The plots of strain cell gauge responses and linearity from biaxial compression for test NML-A06 are shown in Figure 7. Figure 8 is a photograph showing the triaxial strain cell bonded in the EX-hole, test NML-A06. On test NML-A05, a biaxial compression test was not conducted because of the rough core surface and an off-centre EX-hole containing the strain cell.

LABORATORY TESTING

The intact core specimens obtained from stress determination holes and the deformation modulus measurement hole (Site A) were tested in the laboratory. Results from compression tests to obtain the elastic constants and uniaxial compressive strengths are summarized in Table 3.

Core specimens were prepared and strain gauged following the standard testing procedure (6,7). Solid cores and overcore specimens with an EX centre hole were loaded axially. Elastic moduli and Poisson's ratios were calculated at 20 to 50% of the ultimate strength. Uniaxial compressive strengths of 53 mm diameter cores were determined with the 60 ton press, and for 137 mm diameter cores, requiring higher failure load, the 2000 ton loading system was used. Figures 9, 10 and 11 illustrate some examples of the post-failure appearance of core specimens subjected to the unconfined uniaxial compression.

In addition, tests were carried out on dolomite core samples obtained from EX holes in overcoring measurements at Site B. A total of 27 Point Load tests were made using the BEMEK portable rock testing unit. The rock testing unit computes uniaxial compressive strength values from the Point Load Index. Uniaxial compressive strength values ranged between 85.7 and 179.4 MPa with a mean value of 141.30 ± 23.8 MPa.

From the pillar stress measurement sites in weak sulphide (orebody) a limited number of intact core samples was available. Some of the core samples were fractured and only one sample was successfully tested. The remaining three core specimens failed at the initial stage of loading along pre-existing fractures.

RESULTS OF GROUND STRESS DETERMINATIONS

The results of stress determinations from biaxial overcoring measurements and the elastic constants used for the stress tensor calculations are given in Table 4. Regarding stress values obtained in the south abutment,

magnitudes of maximum and minimum secondary principal stresses are relatively consistent with directions of maximum stress ranging from 17 to 39° from vertical. Results from pillar stress determinations are not conclusive because of a too limited number of reliable measurements. However, test NML-P02 in the pillar indicates that the maximum stress acts near-vertical.

Table 5 provides the results of stress determinations from triaxial overcoring measurements. The stress components and the direction of principal compressive stresses are related to the mine coordinate system (Mine grid north - Geographic north). The orientations of the principal stresses are illustrated in an equal area net, lower hemisphere projection, in Figure 12. Comparing the triaxial tests, the stress magnitude varies, but a close agreement exists in the orientation of the principal compressive stresses. The overall orientation for the maximum (σ_1), intermediate (σ_2) and the minimum (σ_3) principal stresses are:

- σ_1 : northeast/horizontal
- σ_2 : southeast/sub-vertical
- σ_3 : northwest/horizontal

The ratios of measured maximum to intermediate and the principal stresses to vertical stress component (σ_v) vary as follows:

σ_1/σ_2	σ_1/σ_v	σ_2/σ_v	$\frac{(\sigma_1+\sigma_2)/2}{\sigma_v}$	$\frac{(\sigma_E+\sigma_N)/2}{\sigma_v}$
2.8-3.6	2.6-3.7	0.9-1.0	1.8-2.4	1.5-2.3

The stress ratios vary over a wide range. For the East portal of the mine, however, the pre-mining vertical stress as well as stresses parallel and perpendicular to the horizontal orebody can best be approximated as follows:

Vertical stress, σ_v : $0.028 \text{ MN/m}^3 \times \text{depth (m)}$ in MPa

Stresses perpendicular to the orebody : $3.2 \times \sigma_v$

Stresses parallel to the orebody : $1.4 \times \sigma_v$.

Comparing the results from biaxial and triaxial measurements in the south abutment, lower stress values were obtained from biaxial measurements.

DETERMINATION OF THE IN SITU DEFORMATION MODULUS

To determine the short-term in situ deformability of rock, dilatometer testing with the Probex-1 dilatometer supplied by RocTest Ltd. was carried out in the orebody (Site A), and in dolomite (Site C). The dilatometer probe operates in N-size boreholes with a diameter of 75 to 78 mm, and has a maximum working pressure of 30 MPa (8). By measuring the total volume change of the probe upon loading, the mean deformation modulus of the rock can be calculated. Figure 13 represents the schematic of the dilatometer system.

Table 6 provides deformation moduli and the equation used for calculation. Pressure-volume curves of individual dilatometer tests are reproduced in Figure 14 and 15. Deformation moduli were determined from the linear parts of the curves in a pressure range between 13.17 to 19.76 MPa. Prior to testing, to establish the representative volume correction factor 'C' (equation in Table 6), several calibration cycles were performed in the steel cylinder at the test site. The volume correction factor used in the equation has a significant effect in calculating the deformation modulus. Typical calibration cycles are plotted in Figure 16.

An alternative approach to estimate the in situ modulus of deformation is based on the geomechanics rock mass rating (RMR) using the equation: $E_M = 2 \text{ RMR} - 100$ (9). Figure 17 represents the relationship between in situ modulus of deformation and geomechanics rock mass rating. For a comparison with the in situ modulus of deformation, values obtained from unconfined compression tests (Table 3) and dilatometer tests (Table 6) are also plotted in Figure 17. The plots indicate a close agreement between the deformation moduli obtained from dilatometer tests and the estimated modulus based on rock mass rating. From the evaluation of the dilatometer tests and the estimated

rock mass rating (Figure 17) deformation moduli in the range of 50-60 GPa for dolomite rock and 20-30 GPa for the orebody are recommended as input parameters for stability analysis.

A combination of underground environmental conditions and an oversize test hole affected the performance of the dilatometer system (e.g., leakage from the O-rings). As a result, tests could not be performed at the sites P37.09/3 and P37.09/1.

STRAIN MONITORING

The purpose of borehole strain monitoring at the mine is to identify ground stress redistributions in underground excavations during pillar extraction. Four CANMET strain monitors were installed in 153 mm diameter horizontal holes drilled for ground stress determinations in the pillar and the south abutment. Each hole was fitted with two strain monitors to measure deformation in vertical and horizontal directions. The monitors were connected to a data logger for continuous recording of strain changes. Location data and calibration factors are given in Table 7. Calibration curves for strain monitors are reproduced in Figures 18 and 19.

It is intended to install additional strain monitoring units in NQ-size holes at sites A and C, and in pillars P37.09/1 and P37.09/3 just prior to pillar recovery. Each hole will be fitted with strain monitors recording vertical and horizontal strains.

The CANMET strain monitoring system (10) uses a vibrating wire strain gauge mounted across a large steel ring which is set with guided wedges into a borehole with a pre-stress of about 10,000 N (2240 lbs). The range is about 450 Hz, and the instrumentation is reliable to 1 Hz = 0.0004 mm. The resolution from the strain along the 153 mm (6 in) diameter hole is 0.0004 mm/153 mm = 2.6×10^{-6} strain. The strain monitoring system is illustrated in Figure 20.

CANMET strain monitors are connected to data acquisition systems to obtain continuous strain records from monitoring sites. Their records will provide the basis for the analysis of mining induced stresses due to pillar recovery.

GROUND MOVEMENT MONITORING

Ground Movement Monitors (GMM) were installed to record pillar expansion and roof (back) separation during pillar recovery. For this purpose ten vertical BQ-holes were drilled into the roof and four horizontal EX-holes, two at each site, into pillars P37.11/2 and P37.09/3. In the pillars, the collars of horizontal holes are about 1.5 m above the floor.

It is intended that GMM be installed in all of the boreholes in the roof with anchors positioned at 3 m from the collar. Readout cables from the units will be fed through the pipes for protection, leading to a control panel which will be located at Site D (Figure 3). The installation of GMM in pillars and roof strata has been completed and initial field data are given in Table 8.

GMM consist of a linearly variable resistor encased in a hexagonal aluminum stock. The case is threaded at one end to a standard rockbolt. A removable centre shaft exits from the other end of the case. The shaft passes through the potentiometer and holds a mounting bracket. GMM have an effective range of 10.16 cm (11). Components of the ground movement monitor and a sketch of the installation procedure are illustrated in Figure 21. Figure 22 shows a typical calibration curve.

VIDEO PROBE INSPECTION OF ROOF HOLES

The vertical BQ holes drilled into the roof for the Ground Movement Monitor installation were inspected with a colour Video Probe. Inspections were carried out from the roof line 6 m into the roof and recorded on video

tape. The video survey of the borehole side wall indicated a competent roof with only some minor horizontal fracturing (cracking) close to the roof line. Boreholes close to the east portal showed occasional horizontal fractures at a depth of 5 to 6 m above the roof line. No significant separations were detected in the roof strata.

Seven holes were inspected (M1, M3, M5, M6, M8, M9, M10). Three holes (M2, M4, M7) were not accessible. Access to the 15 to 20 m high roof was provided with a giraffe which had a basket big enough to accommodate the equipment and two persons for operating the Video Probe unit and holding the rods. For borehole inspection, the video probe was fed through aluminum rods for centering and guided into the roof boreholes.

The components of the Video Probe system are illustrated in Figure 23. The Probe (fibre optic system) has a length of 15.2 m, and the camera head and cable are 10.2 mm O.D. The unit provides axial as well as side viewing. On viewing the video display, the observer needs to consider that the linear magnification is about 5 fold, and areas are magnified 28 times. Side viewing with a 13 inch TV monitor provided a magnification of approximately 3.4 times the 60 mm diameter boreholes. Information on inspected roof boreholes is stored on video tape No. 1.

CONCLUSIONS

The first phase of site investigations at the Nanisivik Mine (East portal) provided the following results:

1. Ground stress determinations in the dolomite of the south abutment showed the maximum stress in a NE-SW/horizontal direction with an average magnitude of 4.5 MPa.
2. In situ rock deformation tests in 76 mm diameter boreholes provided an elastic modulus of 21.4 GPa for the orebody and 60.7 GPa for the surrounding dolomite.

3. Inspection of roof strata up to 6.60 m above the roof line indicated sound roof conditions except for minor blast damage close to the roof line and some minor fractures as the portal is approached.
4. Instrumentation for long-term monitoring of pillar deformation and roof stability has been installed to assess its performance under arctic conditions and to provide advance warning of significant ground movements which may affect ground stability during pillar extraction.

Based on the above findings, numerical modelling will be carried out to identify the effect of pillar removal on roof and general mine stability.

ACKNOWLEDGEMENT

The cooperation of the staff of Nanisivik Mine Ltd. is very much appreciated, especially that of A. Boudreault who provided essential assistance during the field work. Dr. S. Vongpaisal, Canadian Mine Technology Laboratory, CANMET, acted as liaison officer on this project.

REFERENCES

1. Herget, G. and Arjang, B., "Field program and preliminary results of the pillar recovery project at Nanisivik Mines Ltd., Northwest Territories"; Division Report MRL 89-64(TR), CANMET, Energy, Mines and Resources Canada; May 1989.
2. Moormann, J.C., "Pillar recovery at Nanisivik"; Internal Report 80-10-03.
3. Bickel, D.L., "Overcoring equipment and techniques used in rock stress determinations"; Information Circular 9013, U.S. Bureau of Mines; 1985.
4. CSIR, "Instrumentation manual for the use of the CSIR triaxial rock stress measuring equipment"; CSIR Report MER/243, MER/4406, MEG/677, Council for Scientific and Industrial Research, Pretoria, South Africa, 1968.

5. Herget, G., Miles, P. and Zawadski, W., "Equipment and procedures to determine ground stresses in a single drill hole"; Division Report MRP/MRL 78-11, CANMET, Energy, Mines and Resources Canada, Sept. 1977.
6. Gyenge, M. and Herget, G., "Laboratory tests for design parameters"; CANMET Report 77-27; Pit Slope Manual, Suppl. 3-2; CANMET, Energy, Mines and Resources Canada; 1977.
7. ISRM, "Suggested methods for determining the uniaxial compressive strength of rock materials and the Point Load Index"; International Society for Rock Mechanics, Commission on Standardization of Laboratory and Field Tests, Document No. 1, October 1972.
8. Rocctest, "Probex-1 dilatometer - instruction manual"; Rocctest Ltd., 665 Pine, St. Lambert, Montreal, Canada.
9. Bieniawski, Z.T., "Determining rock mass deformability: experience from case histories"; Int J Rock Mech & Geomech Abstr, vol. 15, pp. 237-247, Pergamon Press, 1978.
10. Herget, G. and Kapeller, F., "Monitoring equipment for mining induced stresses at Niobec Mine"; CIM Bull, vol. 82, No. 927, July 1989.
11. Udd, J.E. and Bharti, S., "Monitoring of stability conditions at Falconbridge's Strathcona Mine"; CIM Bull, vol. 80, No. 908, Dec. 1987.
12. Welch Allyn Inc., "Video probe 2000 instruction manual"; Welch Allyn Inc., Skaneateles Falls, New York, 1988.

Table 1 - Diametric deformations from USBM meter measurements

Location	Rock Type	Test No.	Depth (m)	Diametric Deformation (μ in.)			Remarks
				Vert. (U_1)	60°L (U_2)	60°R (U_3)	
Pillar (P37.10/2)	Massive sulphide	NML-PO2	4.40	-56	268	-8	Intact overcore
	ore	NML-PO3	4.80	-19	-45	-40	Core fractured.
		NML-PO4	5.50	-	-	-	Gauge rotated during O/C process.
South abutment (Site B)	Dolomite	NML-A01	3.45	15	20	86	Intact overcore
		NML-A02	4.80	-7	69	25	" "
		NML-A03	5.45	25	48	15	" "
		NML-A04	6.30	14	61	-7	" "

Note: Test NML-PO1 with CSIR-triaxial strain cell at 4.0 m not successful.

Borehole bearing/dip: $120^\circ/+5^\circ$.

Calibration factors:	NML-PO2 to NML-PO4:	NML-A01 to NML-A04:
USBM meter	$U_1=119.04$ (μ in.)	$U_1=115.75$ (μ in.)
	$U_2=130.96$	$U_2=125.15$
	$U_3=115.77$	$U_3=120.65$

U_1 in test holes is oriented within the vertical plane through the axis of the hole.

Table 2 - Elastic strain recovery from CSIR triaxial strain cell measurements

Location	Rock Type	Test No. (Hole No.)	Depth (m)	Gauge No:			Elastic Strain Recovery ($\mu\epsilon$)												SI* Rosette No.		
				1	2	3	4	5	6	7	8	9	10	11	12	1	2	3			
South abutment (Site B)	Dolomite	NML-A05 (1)	6.75	48	40	76	85	27	20	65	69	20	78	55	-10	1	3	7			
	Dolomite	NML-A06 (2)	3.40	42	68	105	83	40	63	122	95	15	unstable readings			4	4	-			

Note: Borehole orientation Hole No. 1: $220^\circ/+5^\circ$, Hole No. 2: $220/+5^\circ$.

*Strain Invariance (sum of strain in two orthogonal directions, $\epsilon_1 + \epsilon_2 = \epsilon_2 + \epsilon_4$).

Table 3 - Determination of Elastic Modulus (E), Poisson's Ratio (ν), and Uniaxial Compressive Strength (UCS) from compression tests.

Test No.	Rock Type (Test Site)	Hgt. (mm)	Dia. (mm)	Ratio H/D	E (GPa)	ν	UCS (MPa)	Failure Load (kN)	Testing Method
NML-SA1	Dolomite	345	137	2.53	57.03	0.26	82.13	1320	Uniaxial compression
NML-SA2	(Site B)	343	"	2.50	53.17	0.23	128.70	1921	"
NML-SA3	"	302	"	2.20	64.07	0.24	153.37	2290	"
NML-SA4	"	295	"	2.15	50.50	0.22	89.40	1355	"
NML-A05	"	341	"	2.49	43.93	0.20	73.70	1100	"
NML-A06	"	-	"	-	42.41	0.23	-	-	Radial compression*
NML-P10	Sulphide (pillar)	315	"	2.30	32.42	0.30	42.00	627	Uniaxial compression
NML-NA1	Sulphide	129	53	2.43	74.23	0.18	75.17	178	Uniaxial compression
NML-NA2	+ quartz	134	"	2.53	64.82	0.12	112.75	267	"
NML-NA3	Stringers (Site A)	132	"	2.50	58.55	0.15	80.10	190	"
NML-EX	Dolomite (Site B)	-	23	-	Mean value: (27 tests)		141.30 ±23.8	-	Point load test**

Note: Elastic modulus and Poisson's ratio obtained at 20 to 56% of failure load.

Uniaxial compressive strength of 137 mm diameter and 53 mm diameter core specimens was determined using 2000 Ton and 60 Ton press, respectively (1 kN = 224.82 lbs).

(*) Values obtained from strain reading of axial and circumferential gauges of CSIR strain cell of the overcored hollow core cylinder in radial loading using Hoek cell.

(**) BEMEK - portable rock tester used.

Table 4 - Magnitudes and directions of secondary principal stresses
from USBM meter measurements

Location	Test No.	Secondary Principal Stresses		Elastic Modulus	ν	Remarks
		Max. Stress P (MPa)	Min. Stress Q (MPa)	Direction of P		
Pillar (P37.10/2)	NML-P02	2.00	-0.38	-26°	0.30	P and Q measured on the plane perpendicular to the axis of the drill hole. Direction of P is measured counter clock- wise from U_1 (U_1 : vertical up)
South abutment (Site B)	NML-A01	1.25	0.35	28°	0.24	
	NML-A02	1.00	0.14	-17°	0.23	
	NML-A03	0.77	0.38	-38°	0.24	
	NML-A04	0.85	0.05	-39°	0.22	
				Ave.	56.26	0.23

Table 5 - Stress components, magnitudes and orientations of the principal stresses from CSIR triaxial strain cell measurements.

Location	Test No.	Stress Components (MPa)					Std. Dev.	Principal Stresses (MPa)		
		σ_{EW}	σ_{NS}	σ_V	τ_{EN}	τ_{NV}		σ_1	σ_2	σ_3
South Abutment (Site B)	NML-A05	2.47 (0.17)	1.45 (0.10)	1.33 (0.10)	1.30 (0.08)	-0.43 (0.08)	0.05	3.50	1.23	0.52
								055/14	280/71	148/13
	NML-A06	3.33 (0.19)	3.25 (0.09)	1.45 (0.10)	2.11 (0.08)	-0.45 (0.11)	0.05	5.45	1.50	1.10
								045/06	304/60	139/29

Note: Stress components and the principal stresses orientation (Bearing/Dip in degrees) related to the mine coordinate system (Mine grid north = Geographic north).

Elastic constants for stress tensor calculations: NML-A05: $E = 43.93$ GPa, $\nu = 0.20$
 NML-A06: $E = 42.41$ GPa, $\nu = 0.23$

Table 6 - In situ deformation moduli from dilatometer measurements

Location	Rock Type	Test No.	Depth from Collar (m)	In Situ Deformation Modulus (GPa)	Remarks
Site A	Sulphide	NQ4-1	1.55	22.60	
	"	NQ4-2	2.50	19.90	
	"	NQ4-3	5.25	21.80	
Site C	Dolomite	NQ3-1	1.50	-	Test not successful
	"	NQ3-2	1.50	-	" " "
	"	NQ3-3	1.50	60.68	---
	"	NQ3-4	2.50	-	Test not successful

Note: Deformation modulus value, E, is calculated based on the solution for an isotropic linear elastic thick hollow cylinder as follows:

$$E = 2(1 + \nu) (V_o + V_m) \frac{1}{\frac{\Delta V}{\Delta P_b} - C}$$

where, ν = Poisson's ratio.

V_o = initial volume of the dilatable membrane, deflated probe.

V_m = mean volume injected into the probe.

$\Delta V = V_1 - V_2$, the difference between the injected volumes corresponding to two points from the linear portion of the loading curve.

$\Delta P_b = 0.955 (P_1 - P_2)$, for a horizontal hole at the same elevation as the probe, 0.955 = pressure correction factor. $P_1 - P_2$ is the difference in pressure between two corresponding points of the loading curve.

$C = a - b$, where "C" is the correction factor for compressibility of the dilatable membrane, "a" is a volume/pressure relationship from calibrating the probe in a steel cylinder, and "b" is the stiffness of the cylinder in terms of volume/pressure.

Table 7 - Location data and calibration factors for strain monitors

Installation Date	Location (Rock Type)	Strain Monitor No.	Depth from Collar (m)	Orientation	Frequency (Hz) Initial	Frequency (Hz) Installed	Calibration Factor (10 ⁻⁴ mm/Hz)
11/5/89	P37.10/2	SMS-122	4.60	Vertical	761	875	4.98
"	(sulphide)	SMS-121	4.40	Horizontal	764	1090	4.31
17/5/89	Site B	SMS-123	4.60	Vertical	737	795	4.76
"	(dolomite)	SMS-124	4.40	Horizontal	754	887	5.25

Table 8 - Location and initial data for ground movement monitors.

Date	Location	Hole No.	Resistance (k Ω)*	
			Red/Green	Red/Black
10/05/1989	P37.11/2	EX.11/2.1	14.62	7.11
"	"	EX.11/2.2	12.15	9.98
"	P37.09/3	EX.09/3.1	20.00	0.20
03/03/1990	Roof	2	18.90	0.91
"	"	3	19.60	0.88
"	"	4	17.38	3.08
"	"	5	20.10	0.01
"	"	6	19.40	1.48
"	"	7	19.80	0.95
"	"	8	19.10	1.03

Note: Ground movement monitors are installed at about 2.4 to 3.6 m from the collar of the boreholes.

(*) Resistance readings between lead wires after installation.
Expansion will increase the resistance readings.

The ground movement monitor was developed by Falconbridge Ltd. (11) and supplied by FREMA R & D Services, Garson, Ontario.

NANISIVIK **QUTAF DASYUQ** **QUTAF DQDQ**
MINES LTD. **NANISIVIK NORTHWEST TERRITORIES, CANADA ADA QXQ**

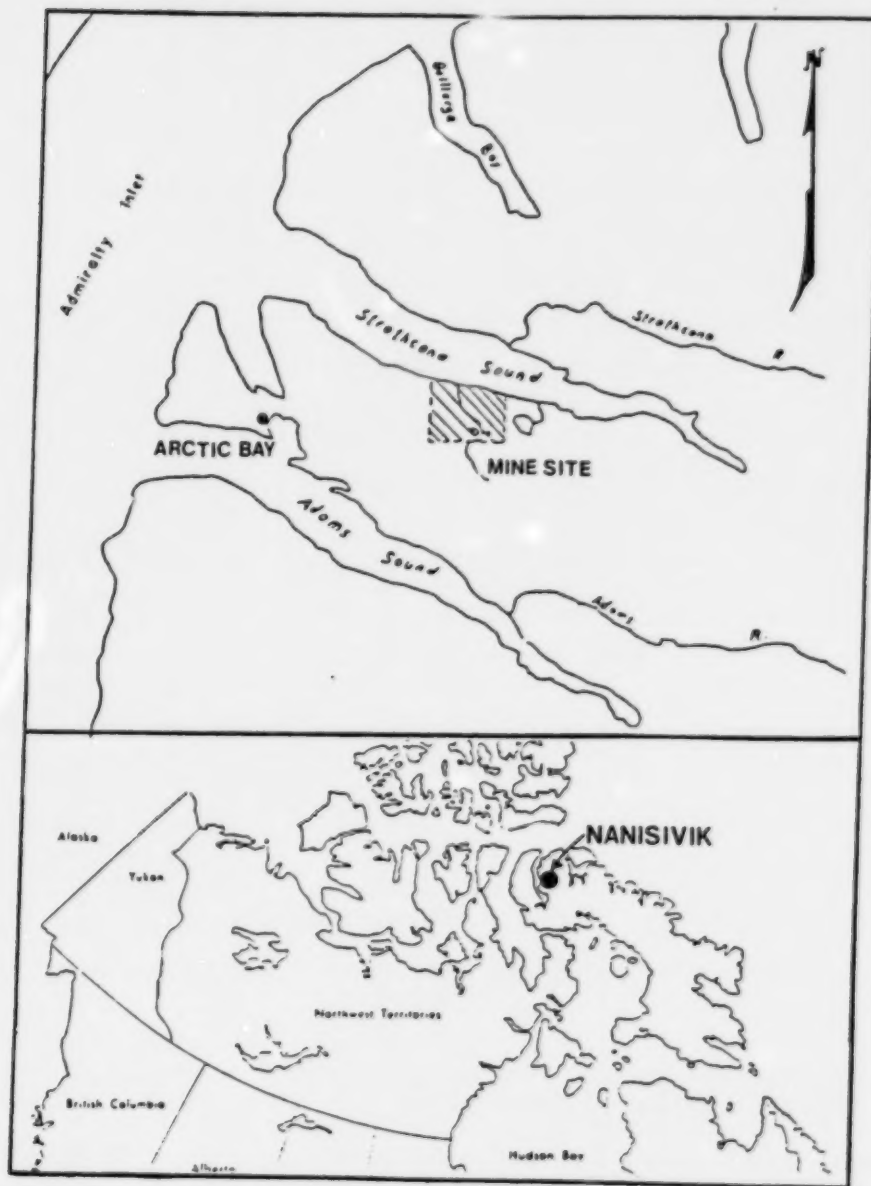


Fig. 1 - Location of Nanisivik Mines Ltd.
 (Latitude $72^{\circ} 08'$, longitude $84^{\circ} 25'$).

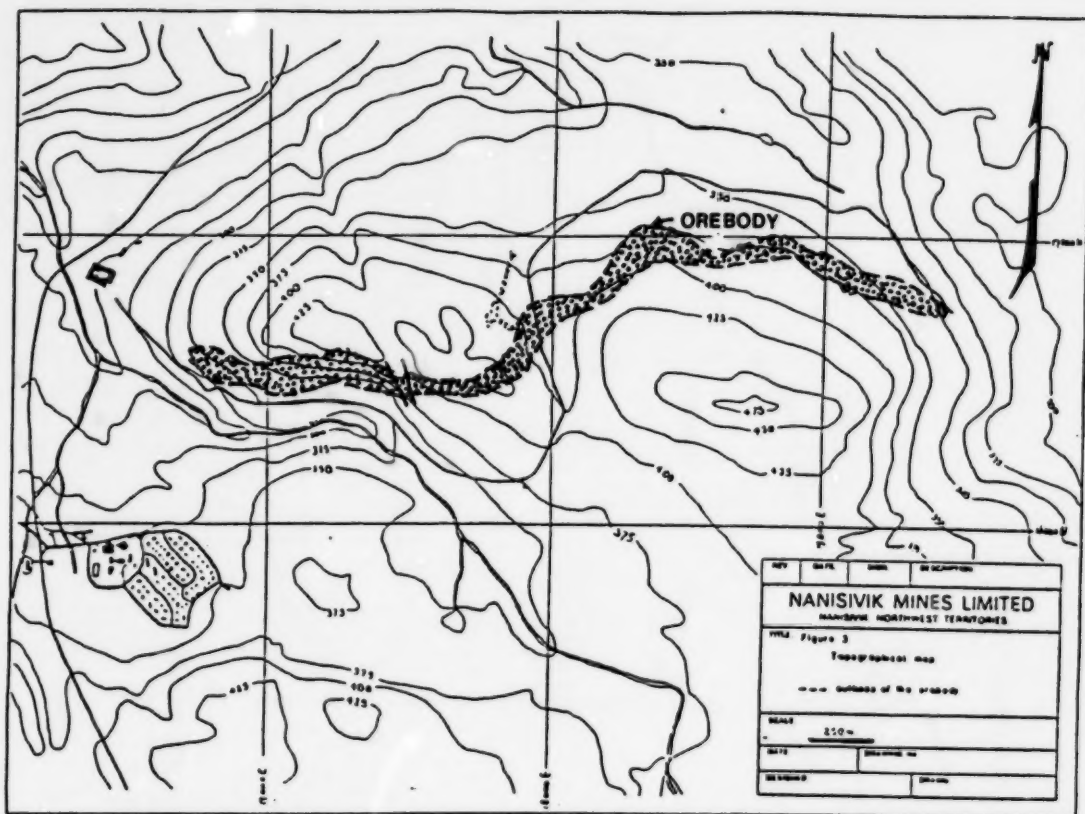


Fig. 2 - Outline of the main orebody.

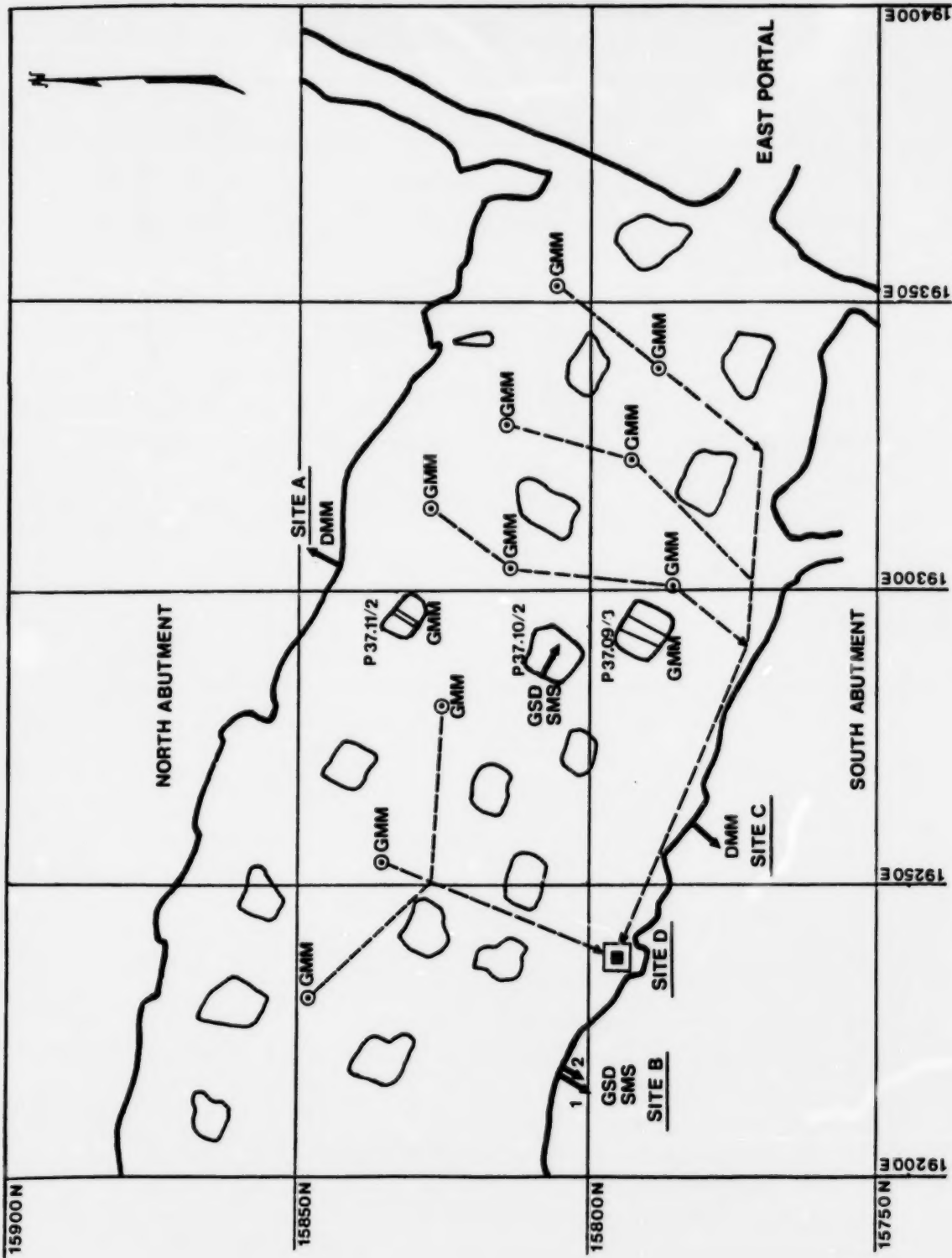


Fig. 3 - Overview of test and instrumentation site.

GSD: Ground Stress Determination
 SMS: Strain Monitoring System
 DMM: Deformation Modulus Measurement
 GMM: Ground Movement Monitor
 ⊙: Roof Boreholes

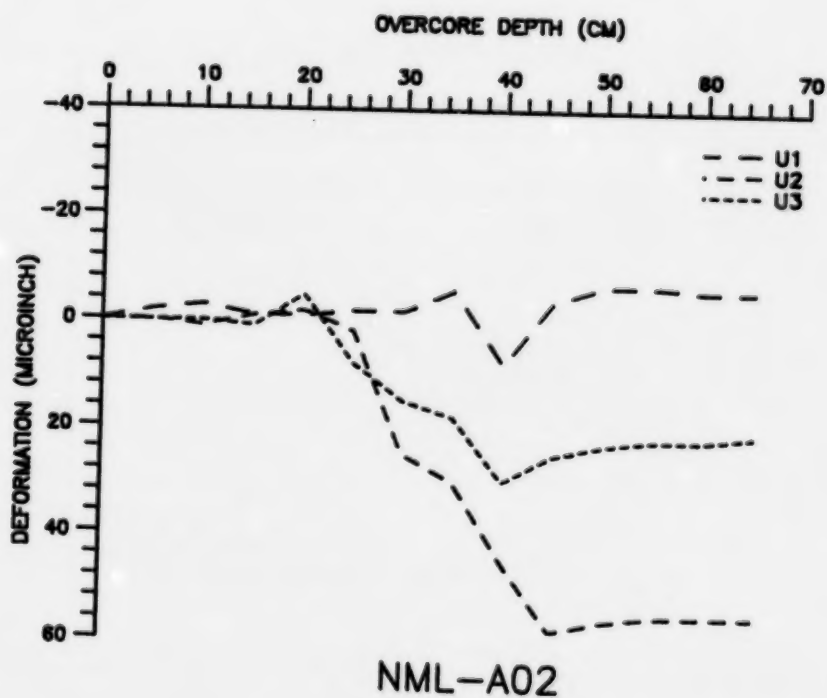
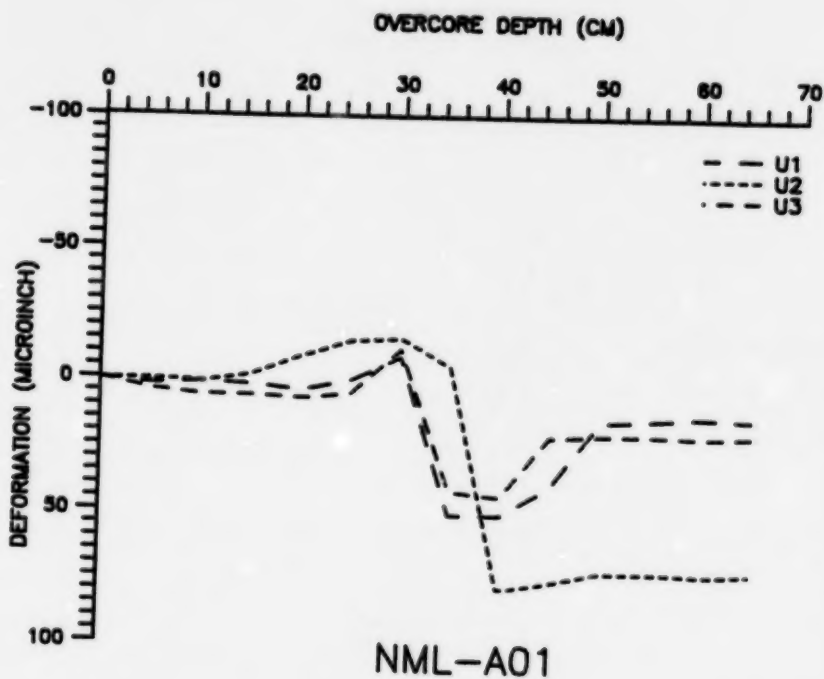


Fig. 4 - Overcoring gauge responses from USBM-meter measurements, NML-A01 and NML-A02.

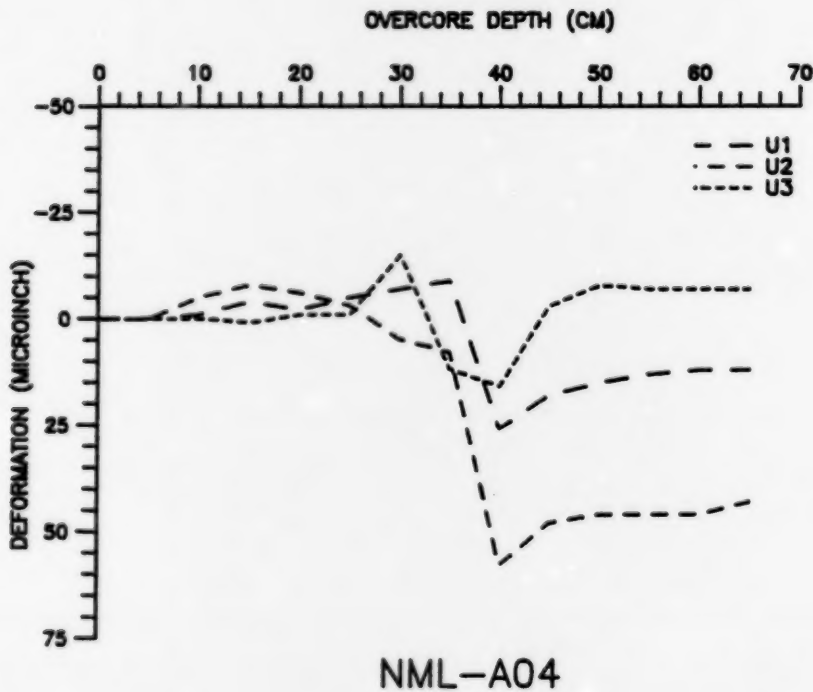
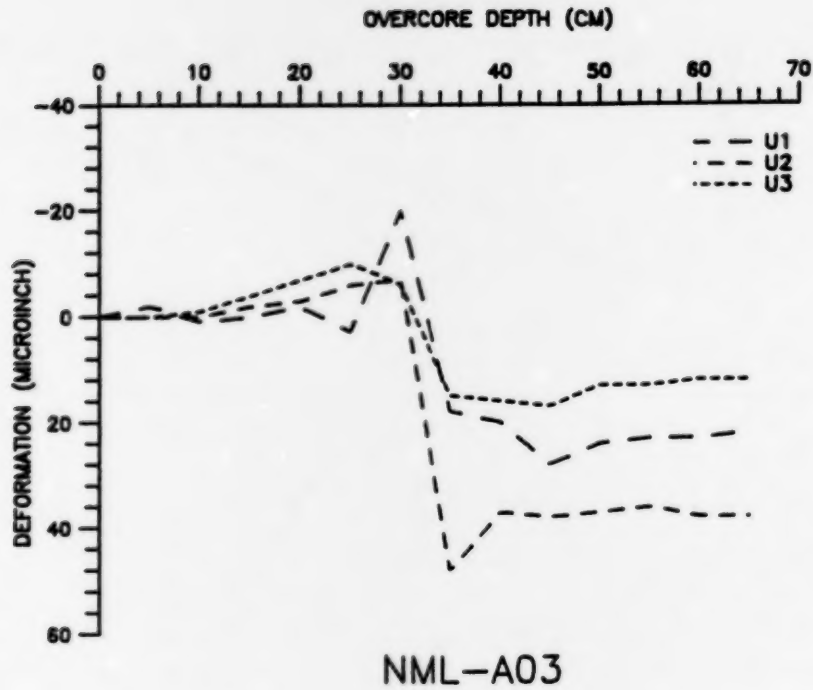


Fig. 5 - Overcoring gauge responses from USBM-meter measurements, NML-A03 and NML-A04.

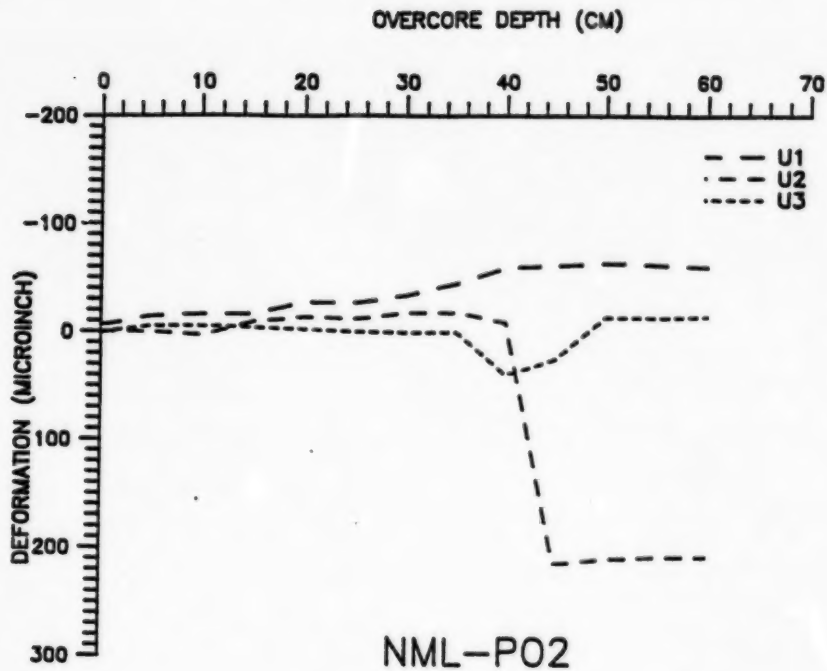


Fig. 6 - Overcoring gauge responses from USBM-meter measurements, NML-P02.

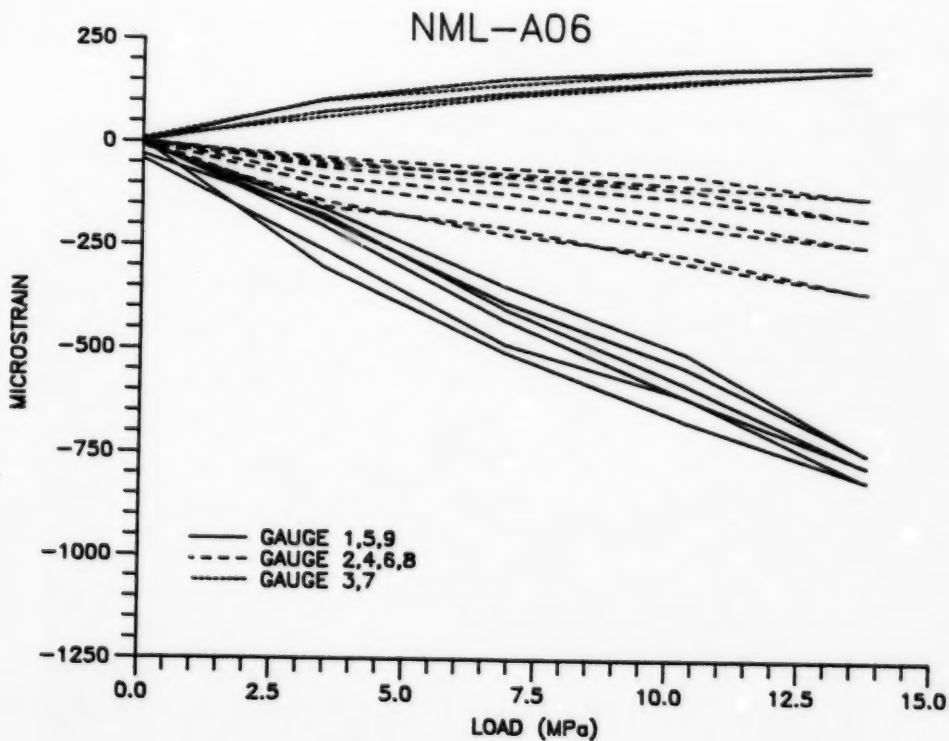


Fig. 7 - Triaxial strain cell gauge response and linearity in biaxial compression, test NML-A06 (gauge 10, 11 and 12 rejected).

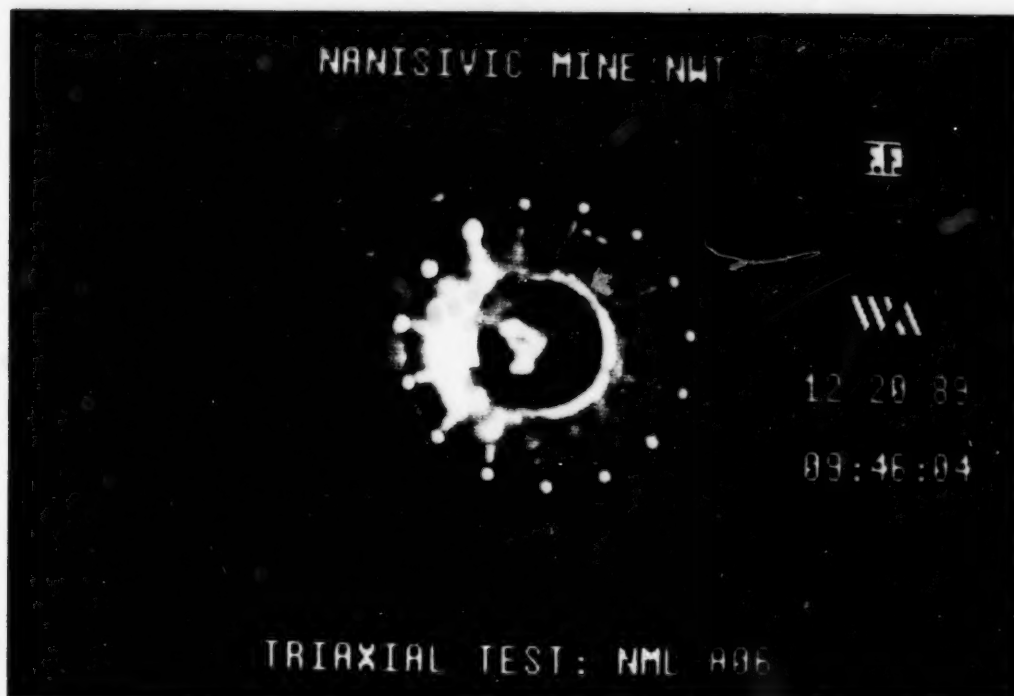


Fig. 8 - Inside view of EX-hole containing a bonded triaxial strain cell (photograph is taken from a TV monitor image with video probe centered in the hole).

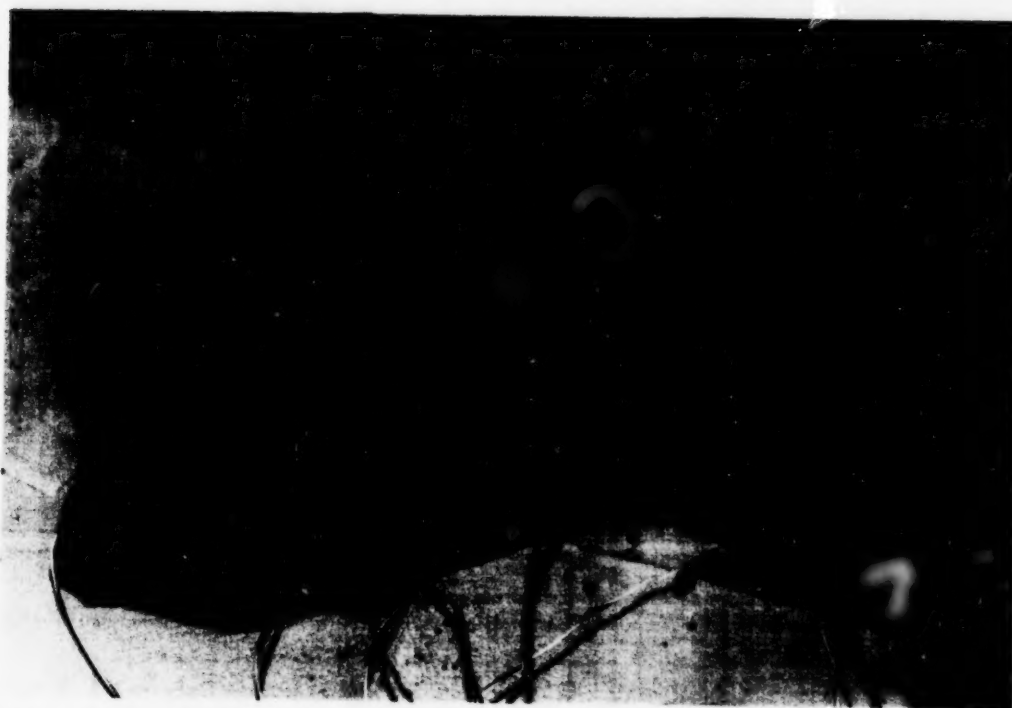


Fig. 9 - Post-failure appearance of overcored specimen, test NML-A05 (core diameter 137 mm, rock type: dolomite).

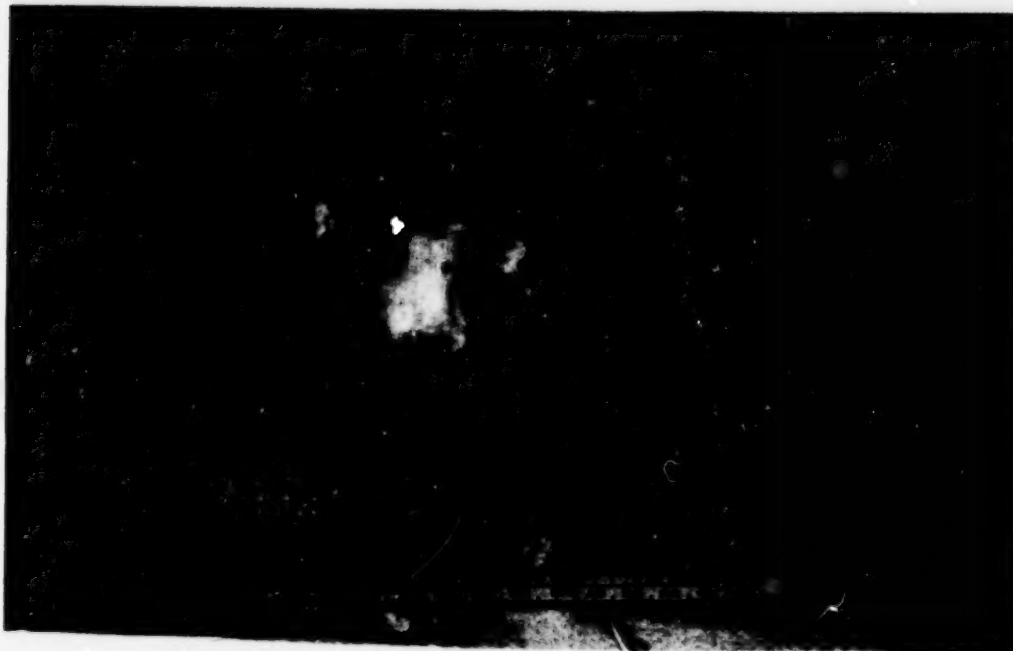


Fig. 10 - Post-failure appearance of 53 mm diameter core specimen of sulphide ore from Site A.

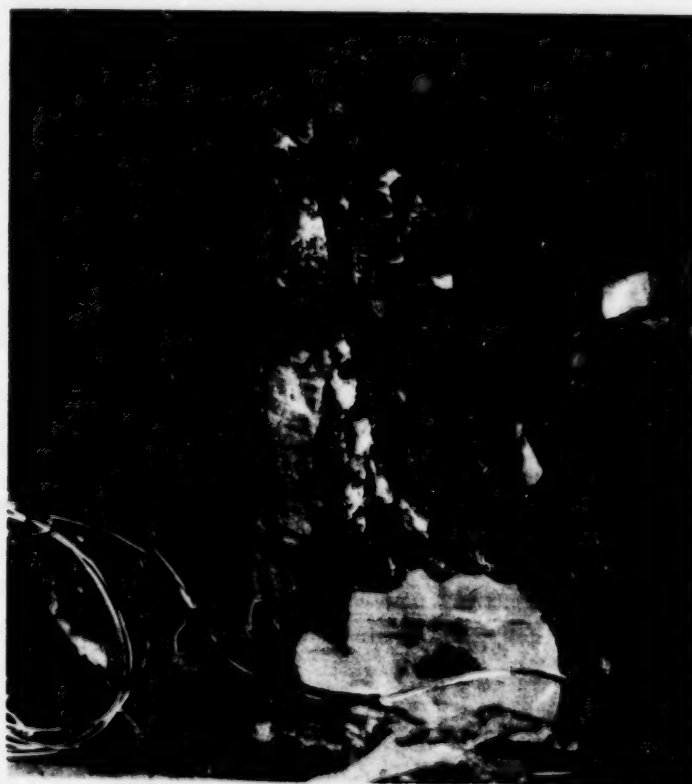


Fig. 11 - Post-failure appearance of a 137 mm diameter core specimen of dolomite from Site B.

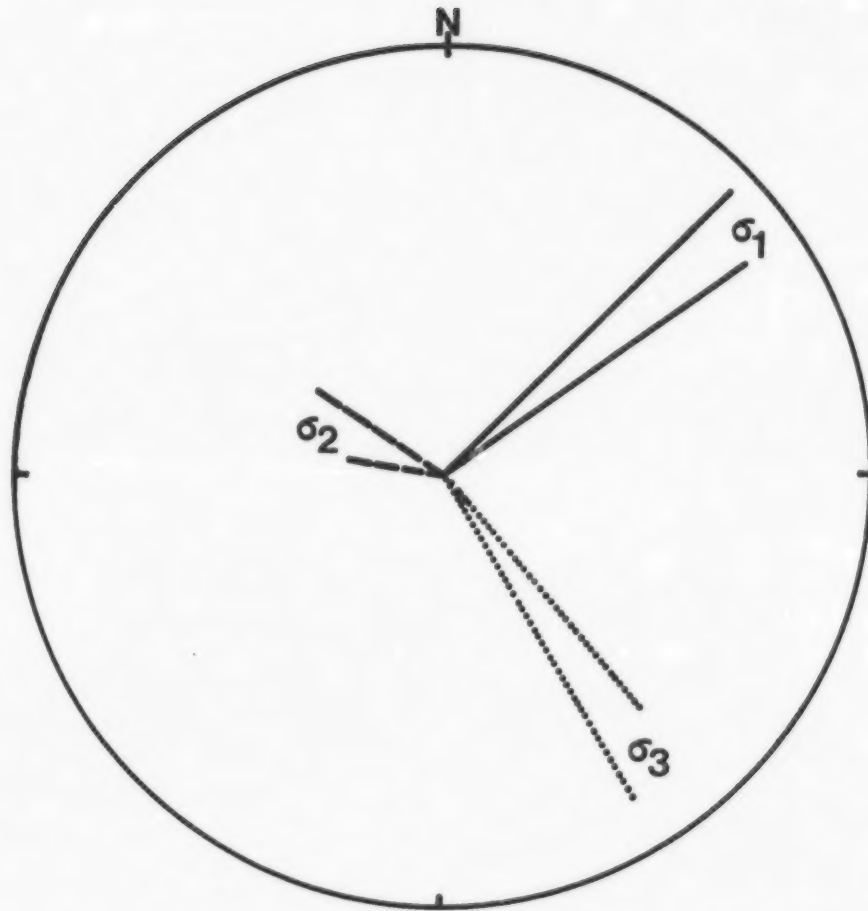


Fig. 12 - Orientation of the principal compressive stresses from Mine Grid north (=Geographic north) in an equal area net, lower hemisphere projection.

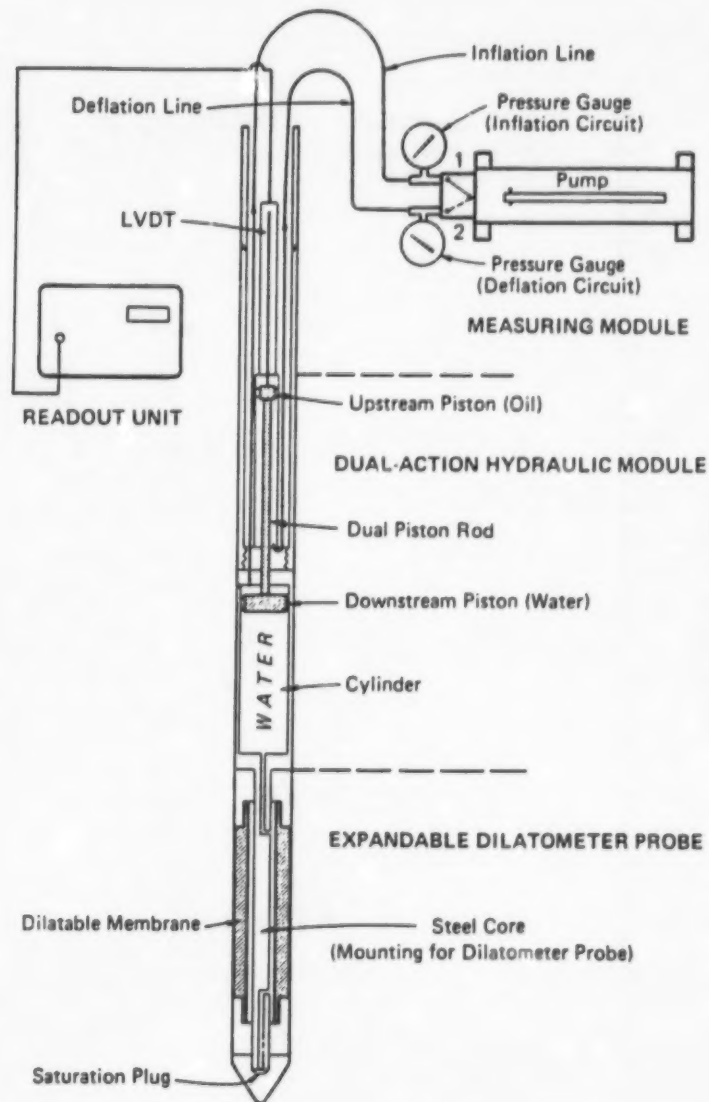


Fig. 13 - Schematic of the PROBEX-1 Dilatometer system (8).

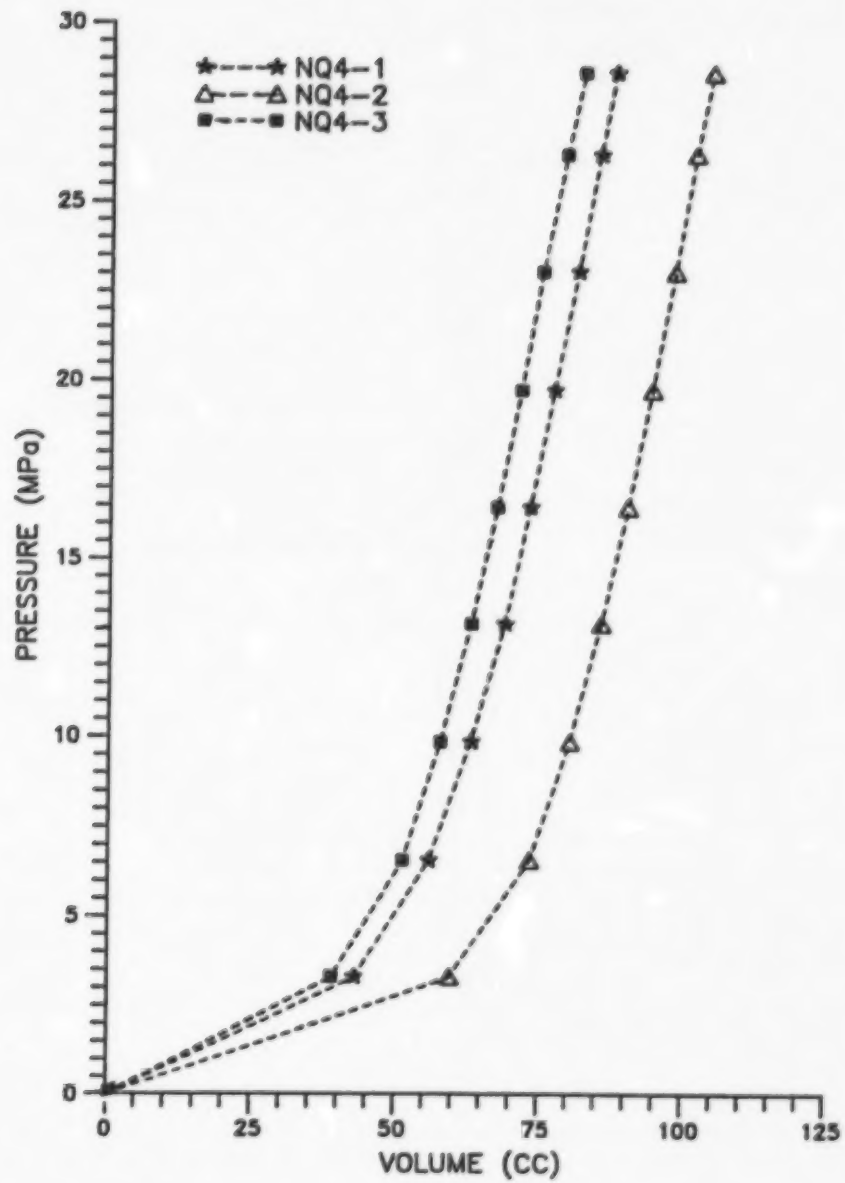


Fig. 14 - Plot of the in situ deformation modulus measurements, Site A.

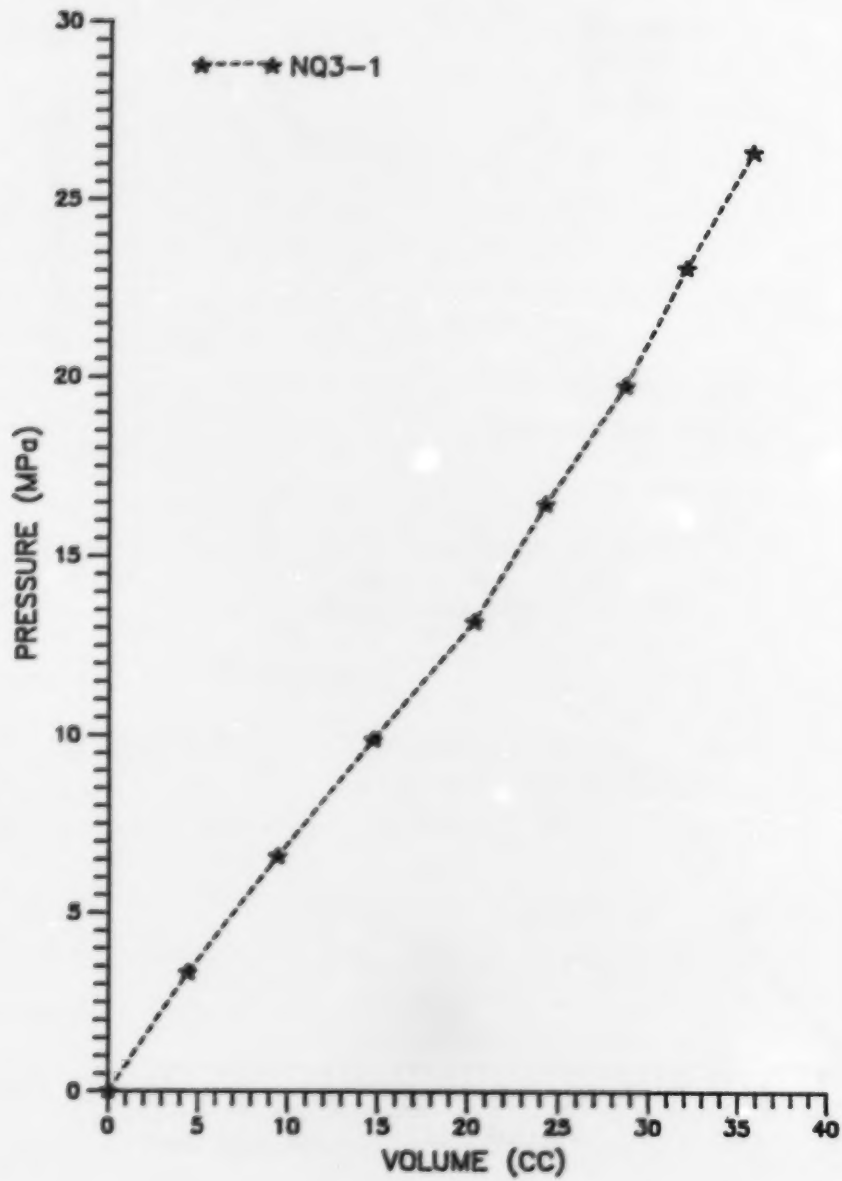


Fig. 15 - Plot of the in situ deformation modulus measurements, Site C.

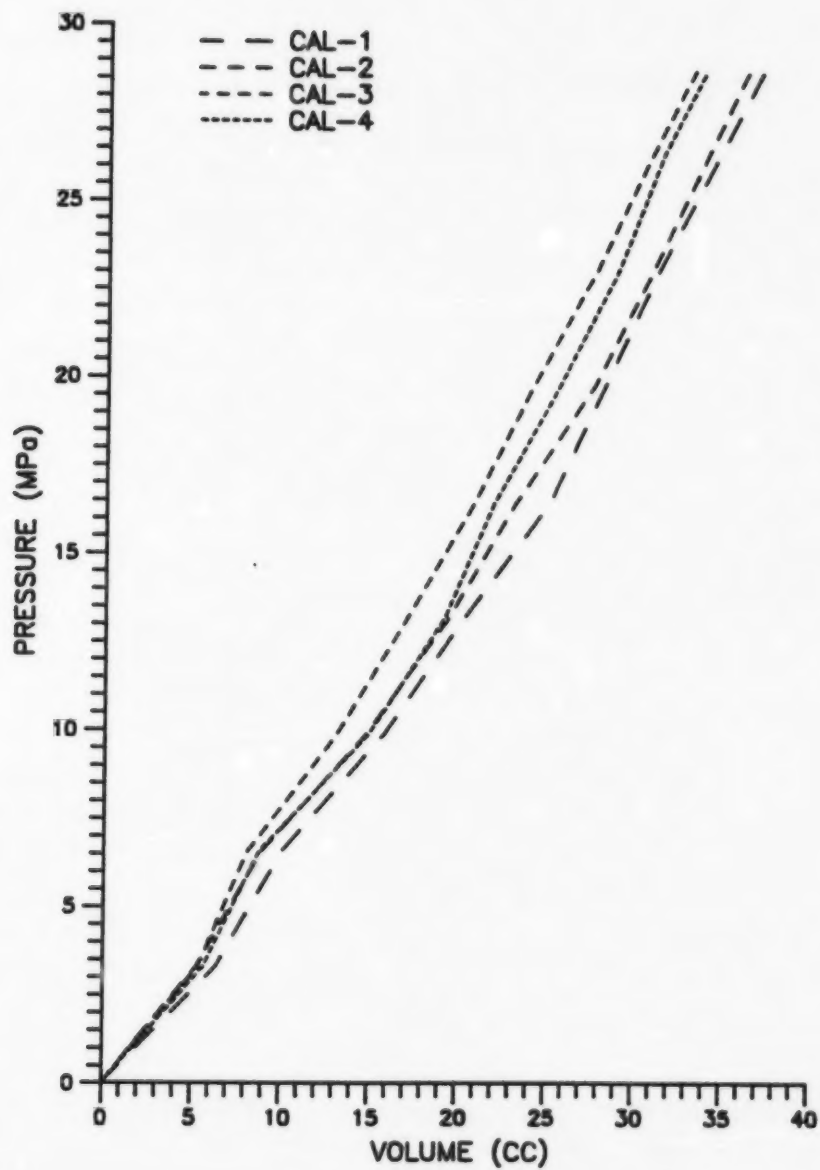
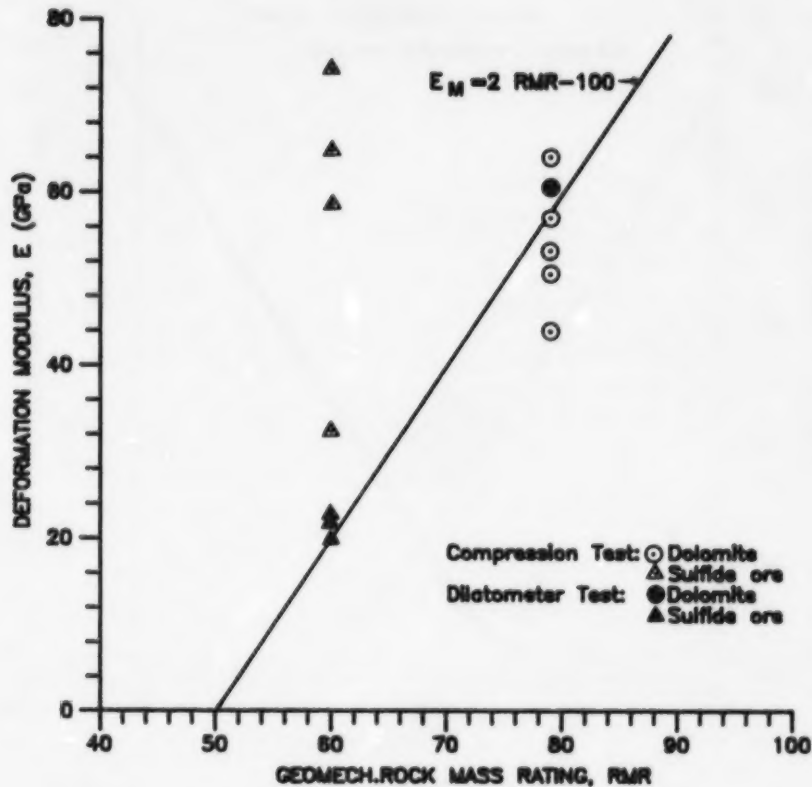


Fig. 16 - Typical calibration cycles of the dilatometer system obtained underground (Site A).

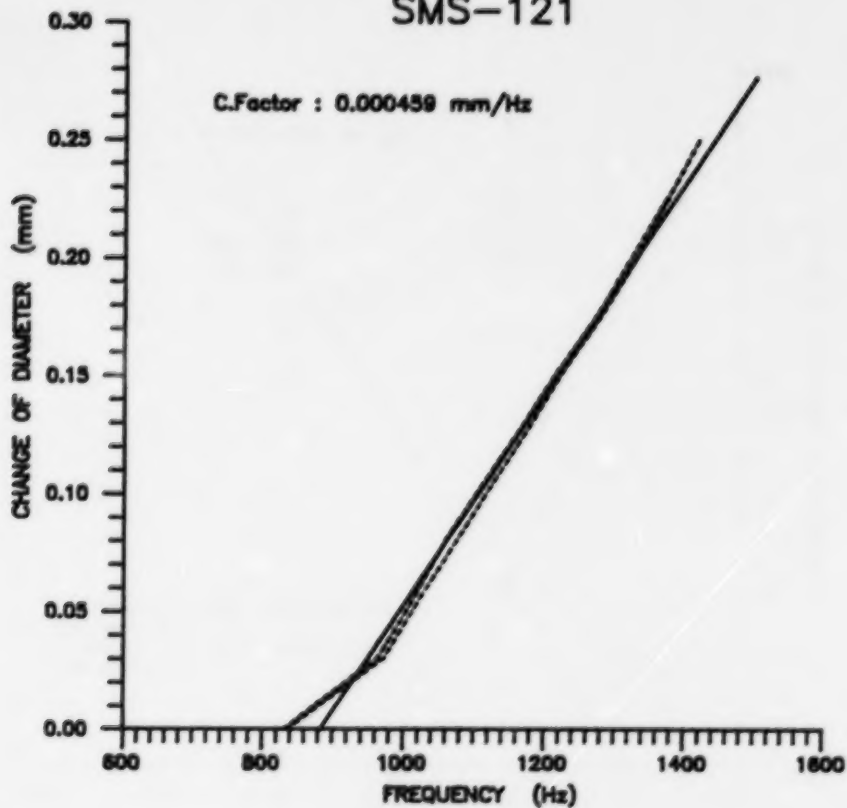


Geomechanics Rock Mass Rating, RMR:

Classification Parameter	Value/Description		Rating	
	Dolomite	Sulphide Ore	Dolomite	Sulphide Ore
1. Strength of intact material	140 MPa	90 MPa	12	7
2. RQD	75-90%	50-75%	17	13
3. Joint spacing	0.3-1 m	50-300 mm	20	10
4. Condition of joints	Slightly rough surfaces		20	20
	Separation <1 mm			
	Hard joint wall rock			
5. Ground water	No inflow/completely dry		10	10
Rock Mass Rating (RMR), total score =			79	60

Fig. 17 - Relationship between in situ modulus and rock mass rating, plots represent moduli determined in Tables 3 and 6.

SMS-121



SMS-122

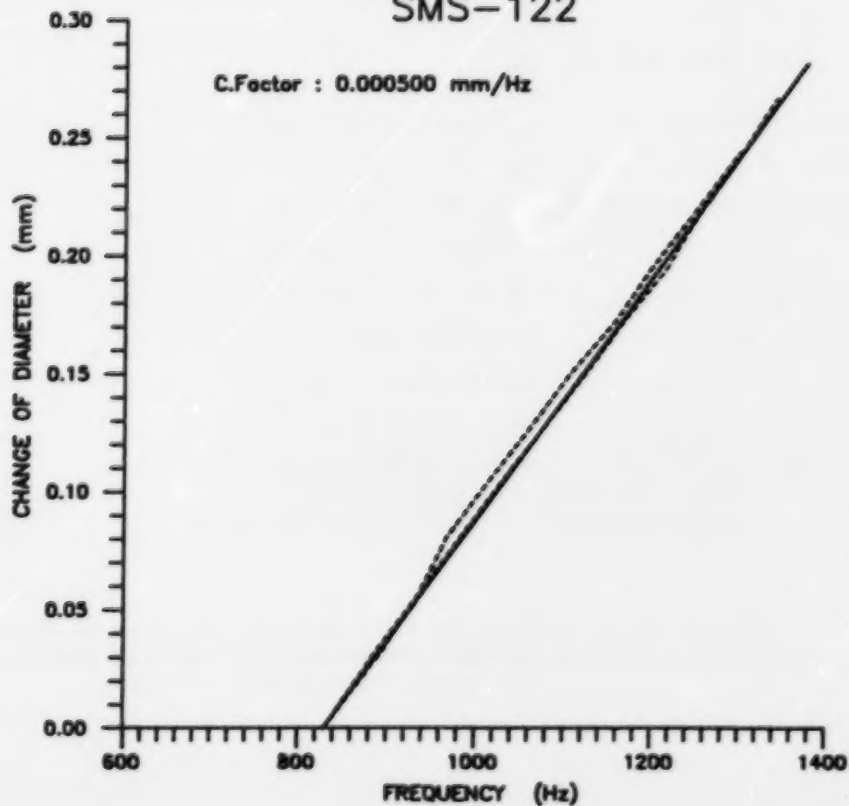
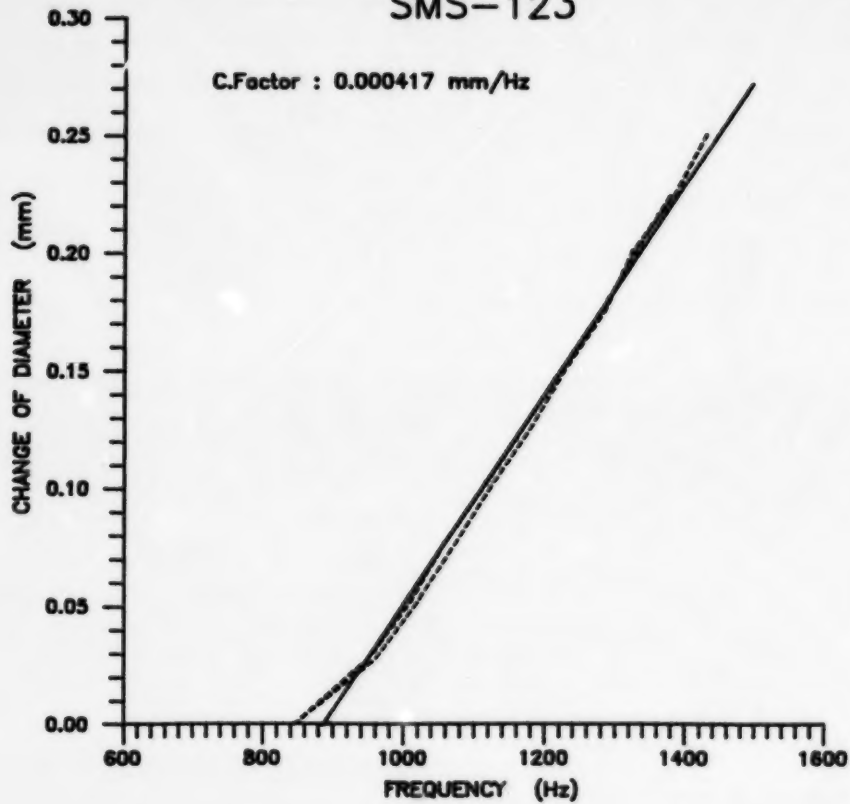


Fig. 18 - Calibration curves for strain monitors SMS-121 and SMS 122.

37
SMS-123



SMS-124

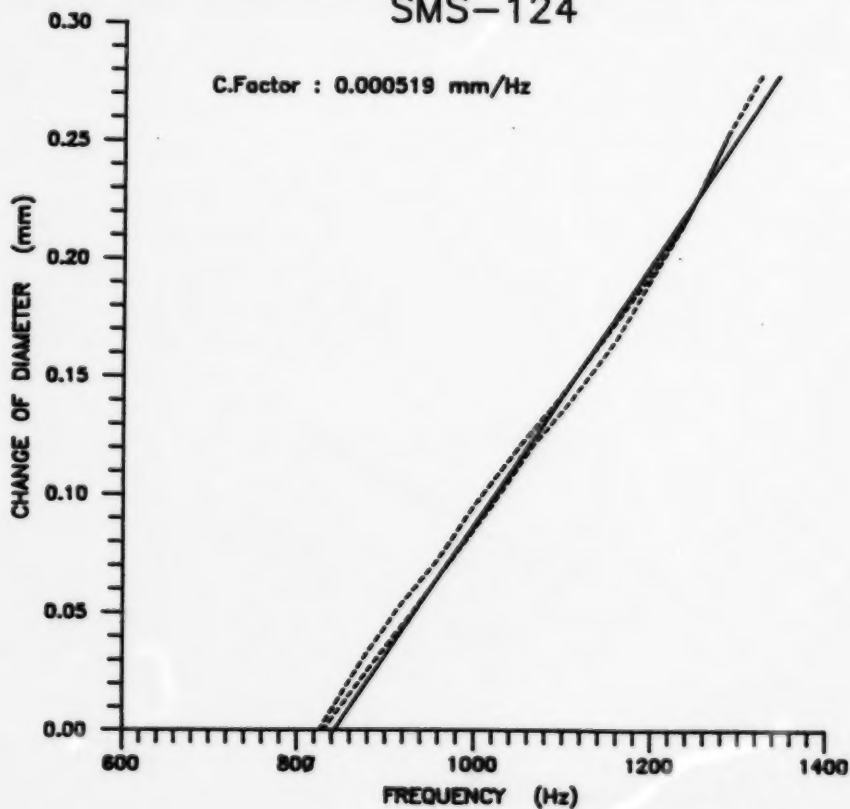


Fig. 19. Calibration curves for strain monitors SMS 123 and SMS 124.

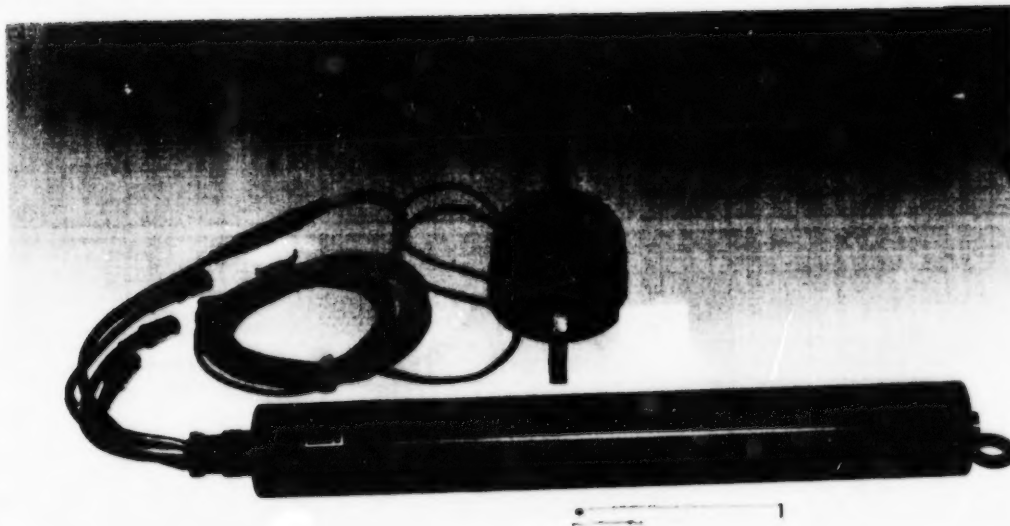
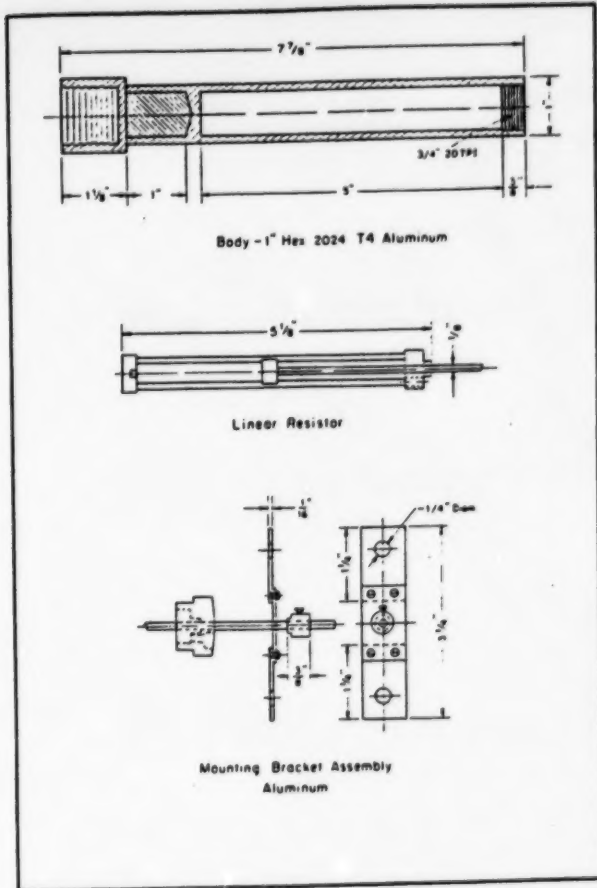
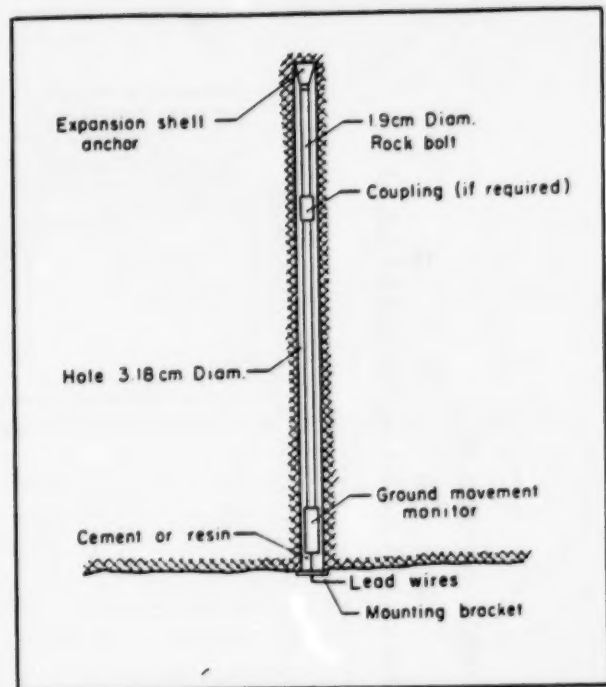


Fig. 20 - CANMET strain monitoring system.



B- Details of GMM components



C- Sketch of GMM installation



A - Ground movement monitor

Fig. 21 - Ground movement monitor, GMM (11).

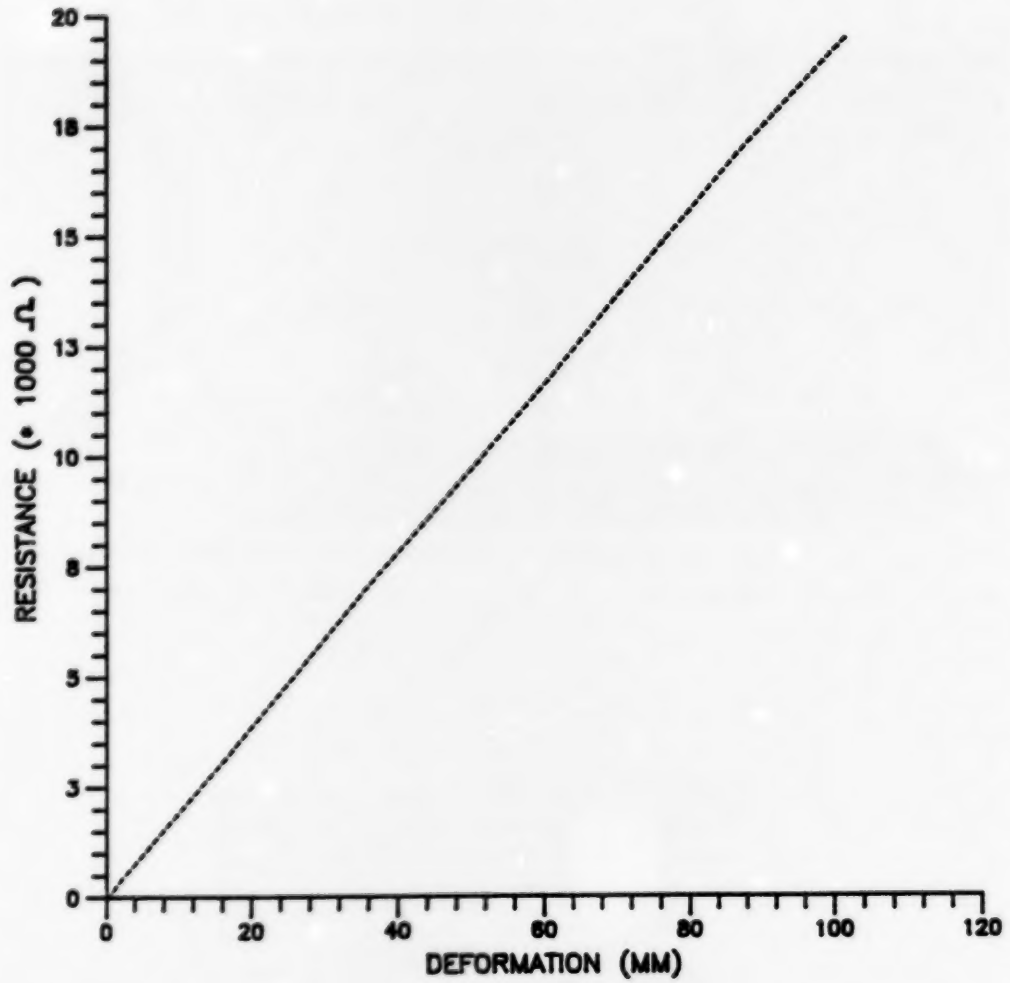


Fig. 22 - Typical calibration plot of a ground movement monitor.

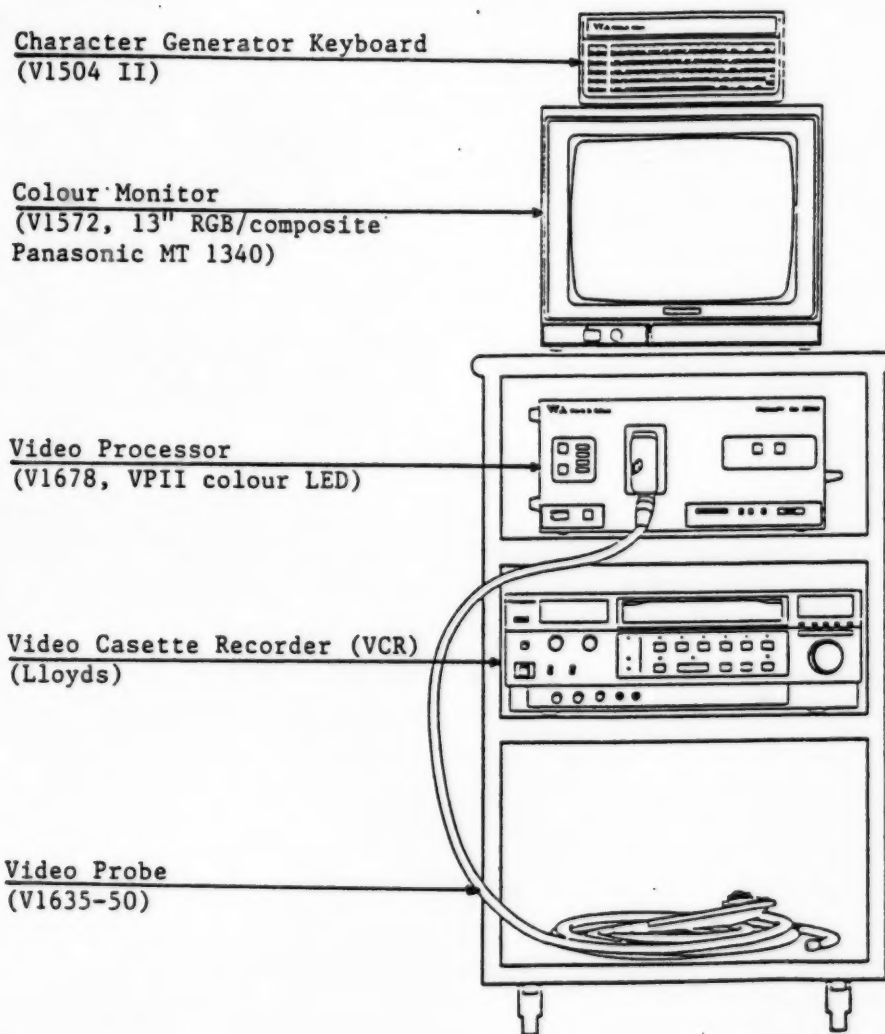


Fig. 23 - Video probe 2000 - system components (12).